Analysis of Alongwind Tall Building Response to Transient Nonstationary Winds

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Abstract: Transient nonstationary extreme winds such as thunderstorm downbursts are responsible for significant structural damage and failures. This study deals with the frequency domain analysis of alongwind tall building response to transient nonstationary winds based on nonstationary random vibration theory. The transient wind fluctuations and associated wind loads are modeled as the sum of deterministic time varying mean and evolutionary random fluctuating components. The alongwind loads on buildings are determined through the approaching winds by using strip theory and taking into account the unsteady force characteristics in terms of aerodynamic admittance and joint acceptance functions. An analysis framework is developed to quantify the time varying mean, evolutionary spectrum, and time varying root-mean-square values of building response. The traditional analysis framework concerning stationary boundary layer winds serves as a special case of this novel framework. Applications of this general framework are addressed to the cases where the mean wind speed is characterized by a time-invariant vertical profile and a single time varying function that also serves as the modulation function for the wind fluctuations. The influence of time varying mean wind speed, mean wind speed vertical profile, and spatial correlation of wind fluctuations on building response is discussed using tall building examples.

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Introduction

Extreme wind events in many parts of the United States and the world are caused by severely convective thunderstorm winds, which are responsible for significant structural damage and failures (Twisdale and Vickery 1992; Holmes 1999; Changnon 2001; Letchford et al. 2001; Letchford et al. 2001; Choi 2004). The flow field created by such an event varies significantly from the traditional atmospheric boundary layer wind flows in terms of its unique mean wind speed vertical profile, rapid time varying mean wind speed, and spatially strongly correlated wind fluctuations (Fujita 1990; Letchford et al. 2001). The transient nonstationary features of thunderstorm winds may markedly influence wind-structure interactions and wind load effects on buildings. Despite this, over the past 40 years, extensive efforts have been made to understand and quantify stationary boundary layer wind effects on structures, and the methodologies developed have formed the basis for current wind loading codes and standards (Davenport 1967, 1995; Piccardo and Solari 2000; Letchford et al. 2001; Chen and Kareem 2004). The understanding of fundamental characteristics of those transient nonstationary wind effects on structures has yet to be developed.

Characterization and modeling of transient winds and their effects on structures through field observations, numerical, and physical simulations have been receiving increasing attention in recent years (Holmes and Oliver 2000; Wood et al. 2001; Chay et al. 2002; Letchford and Chay 2002; Gast and Schroeder 2003; Haan et al. 2003; 2006; Hangan et al. 2003; Choi 2004; Chay et al. 2006; Kareem et al. 2006). The transient nonstationary feature of extreme winds is not limited to thunderstorm downbursts, but also observed in typhoon-induced winds (Kawai 2000; Xu and Chen 2004; Wang and Kareem 2005). Concerning the quantification of nonstationary wind load effects on structures, Kawai (2000) addressed the influence of time varying typhoon mean wind speed on structural response based on a so-called quasi-stationary model, which was considered applicable when the structural natural period is sufficiently smaller than the time scale of the variation of the mean wind speed. Choi and Hidayat (2002) analyzed dynamic response of single-degree-of-freedom (SDOF) structures to thunderstorm winds through spectral analysis using measured wind speed time histories. Chen and Letchford (2004a,b) conducted response time history analysis of tall buildings to simulated and measured thunderstorm downbursts. Holmes et al. (2005) quantified the dynamic response spectrum of SDOF structures for a given thunderstorm wind time history record, following the concept similar to the earthquake response spectrum. The random vibration theory has widely been employed for quantifying structural response to stationary winds. However, the theory concerning structural response to nonstationary excitations (Lin and Cai 1995; Lutes and Sarkani 2004) has not yet been adopted in wind engineering applications, while it has many applications in earthquake engineering for analyzing structural response under nonstationary ground excitations (Conte and Peng 1997; Michaelov et al. 1999).

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fluctuations and associated wind loads are modeled as the sum of deterministic time varying mean and evolutionary random fluctuating components. The alongwind loads on buildings are determined through the approaching winds by using strip theory and taking into account the unsteady force characteristics in terms of aerodynamic admittance and joint acceptance functions. An analysis framework is developed to quantify the time varying mean, evolutionary spectrum, and time varying root-mean-square (RMS) values of building response. Applications of this general framework are addressed to the cases where the mean wind speed is characterized by a time-invariant vertical profile and a single time varying function that also serves as the modulation function for the wind fluctuations. The influence of time varying mean wind speed, mean wind speed vertical profile, and spatial correlation of wind fluctuations on building response is discussed using tall building examples.

General Analysis Framework

Consider the alongwind response of a tall building under a transient nonstationary wind excitation. The wind speed at elevation \( z \) above the ground \( U(z,t) \) can be decomposed into the deterministic time varying mean component \( \bar{U}(z,t) \) and the random fluctuating component \( u'(z,t) \)

\[
U(z,t) = \bar{U}(z,t) + u'(z,t)
\]

(1)

The fluctuating component \( u'(z,t) \) is regarded as a zero mean evolutionary random process, which is modeled as a zero mean stationary process \( u(z,t) \) modulated with a complex-valued deterministic modulation function \( g(z,\omega,t) \). \( u'(z,t) \) and \( u(z,t) \) are expressed in the general form of a Fourier-Stieltjes integral as

\[
u'(z,t) = \int_{-\infty}^{\infty} g(z,\omega,t)e^{i\omega t}d\Theta(z,\omega)
\]

(2)

\[
u(z,t) = \int_{-\infty}^{\infty} e^{i\omega t}d\Theta(z,\omega)
\]

(3)

where \( i = \sqrt{-1}; \omega = \text{circular frequency}; \ g(z, -\omega ,t) = g^*(z, \omega, t) \) (where * denotes complex conjugate); \( d\Theta(z,\omega) = \text{complex-valued zero mean orthogonal increment random process} \)

\[
E[d\Theta(z,\omega)] = 0, \quad d\Theta(z,\omega) = d\Theta^*(z, -\omega)
\]

(4)

\[
E[d\Theta^*(z_1,\omega_1)d\Theta(z_2,\omega_2)] = S_\Theta(z_1, z_2, \omega_1)\delta(\omega_1 - \omega_2)d\omega_1
\]

(5)

where \( E[\ldots] \) denotes the expectation or ensemble average; \( \delta[\ldots] \) represents the Dirac delta function; \( S_\Theta(z,\omega) = S_\Theta(z,\omega) \) and \( S_\Theta(z_1, z_2, \omega) \) are the power spectral density (PSD) function of \( u(z,t) \) and cross-spectrum for \( u(z_1,t) \) and \( u(z_2,t) \).

The evolutionary PSD (EPSS) of \( u'(z,t) \), and the evolutionary cross-spectrum for \( u'(z_1,t) \) and \( u'(z_2,t) \) are given as

\[
S_{u'}(z,\omega,t) = |g(z,\omega,t)|^2S_\Theta(z,\omega)
\]

(6)

\[
S_{u'}(z_1,z_2,\omega,t) = g^*(z_1,\omega,t)g(z_2,\omega,t)S_\Theta(z_1,z_2,\omega)
\]

(7)

Based on strip theory, the alongwind force per unit building height at the elevation \( z \) above the ground \( P(z,t) \) is expressed in terms of the approaching wind as

\[
P(z,t) = \bar{P}(z,t) + P'(z,t)
\]

(8)

where \( \bar{P}(z,t) \) and \( P'(z,t) \) are deterministic time varying mean and random fluctuating components of \( P(z,t) \), respectively; \( p = \text{air density}; \ B = \text{building width}; \ \ Cd = \text{drag coefficient}; \) and \( \chi_d(\omega) = \text{complex-valued aerodynamic admittance function}, \) which represents the transfer function between approaching wind and wind force. It is noted that the square of the magnitude of \( \chi_d(\omega) \), i.e., \( |\chi_d(\omega)|^2 \), is also often referred to as aerodynamic admittance function in practice.

The alongwind building response is generally dominated by its fundamental modal response, and the higher mode contributions can be assumed to be negligible. The generalized force \( \bar{Q}(t) \) associated with the fundamental mode shape, \( \phi(z) = z/H)^B \) (where \( H \) is the building height; and \( \beta \) is the mode shape exponent ranging between 1.0 and 1.5 for typical tall buildings), is given as

\[
\bar{Q}(t) = \bar{Q}(t) + Q'(t)
\]

(9)

\[
\bar{Q}(t) = \int_0^H \bar{P}(z,t) \left( \frac{z}{H} \right)^\beta dz = \int_0^H \bar{A}(z,t)dz
\]

(10)

\[
Q'(t) = \int_0^H P'(z,t) \left( \frac{z}{H} \right)^\beta dz = \int_0^\infty A(z,\omega,t)e^{i\omega t}d\Theta(z,\omega)dz
\]

(11)

\[
\bar{A}(z,t) = (0.5pCdB) \left( \frac{z}{H} \right)^\beta \bar{U}(z,t)
\]

(12)

\[
A(z,\omega,t) = (pCdB) \left( \frac{z}{H} \right) \bar{U}(z,t)\chi_d(\omega)g(z,\omega,t)
\]

(13)

Consequently, the generalized displacement \( q(t) \) is expressed as follows in terms of its deterministic time varying mean and random fluctuating components, which are the responses under \( \bar{Q}(t) \) and \( Q'(t) \), respectively

\[
q(t) = \tilde{q}(t) + q'(t)
\]

(14)

\[
\tilde{q}(t) = \int_0^t h(t - \tau)\bar{Q}(\tau) d\tau = \int_0^H \tilde{G}(z,t)dz
\]

(15)

\[
q'(t) = \int_0^t h(t - \tau)Q'(\tau) d\tau = \int_0^\infty \int_{-\infty}^{\infty} G(z,\omega,t)e^{i\omega t}d\Theta(z,\omega)dz
\]

(16)
\[ h(t) = \frac{1}{m_1} e^{-\xi_1 \omega_1 t} \sin(\omega_1 t) \quad (t \geq 0) \] (20)

where \( m_1 \), \( \xi_1 \), and \( \omega_1 \) = generalized mass, damping ratio, and natural circular frequency of the fundamental mode; and \( \omega_{D1} = \omega_1 / \sqrt{1 - \xi_1^2} \) = damped natural circular frequency.

It is worthy noting that \( \tilde{G}(z, t) \) can be interpreted as the fundamental mode response to the excitation \( \tilde{A}(z, t) \). Similarly, \( \tilde{G}(z, t) \) is \( e^{iwt} \) multiplied by the fundamental mode response to the excitation \( A(z, \omega, t) e^{iwt} \). They can be generally quantified by a step-by-step integration method such as Newmark’s method or closed-form formulations in some limited cases.

The autocorrelation function of \( q'(t) \) is expressed as
\[
R_{q'}(t, \tau) = E[q'(t)q'(t + \tau)]
= \int_{-\infty}^{H} \int_{0}^{H} G^*(z_1, \omega, t) G(z_2, \omega, t)
+ \tau) S_u(z_1, z_2, \omega) e^{i\omega t} dz_1 dz_2 d\omega
\] (21)

which, for \( \tau=0 \), becomes
\[
R_{q'}(t, 0) = R_{q'}(t)
= \int_{-\infty}^{H} \int_{0}^{H} G^*(z_1, \omega, t) G(z_1, z_2, \omega) S_u(z_1, z_2, \omega) dz_1 dz_2
\] (22)

Accordingly, the EPSD of \( q'(t) \) is given as
\[
S_{q'}(z, \omega) = \int_{0}^{H} \int_{0}^{H} G^*(z, \omega, t) G(z, z, \omega) S_u(z_1, z_2, \omega) dz_1 dz_2
\] (23)

Once the generalized displacement is quantified, any response of interest (e.g., shear force and bending moment), \( r(t) = \Gamma q(t) \) \( = \tilde{r}(t) + \tilde{r}'(t) \), are then determined as
\[
\tilde{r}(t) = \tilde{r} / \Gamma \quad R_{r}(t, \tau) = \Gamma^2 R_{q'}(t, \tau) \quad S_r(\omega, t) = \Gamma^2 S_{q'}(\omega, t)
\] (24)

where \( \Gamma = \) modal participation coefficient for response \( r(t) \); \( \tilde{r}(t) \) and \( \tilde{r}'(t) \) = deterministic and random components of \( r(t) \); and \( R_{r}(t, \tau) \) and \( S_r(\omega, t) \) = autocorrelation function and EPSD of \( r'(t) \).

Similarly, the \( p \)th time derivative of the generalized displacement, i.e., \( q^{(p)}(t) = \tilde{q}^{(p)}(t) + q^{(p)}(t) \) can be calculated. The deterministic component \( \tilde{q}^{(p)}(t) \) is directly determined though the \( p \)th time derivative of \( \tilde{q}(t) \), while the random component \( q^{(p)}(t) \) can be expressed as
\[
q^{(p)}(t) = \int_{0}^{H} \int_{-\infty}^{H} \tilde{G}^{(p)}(z, \omega, t) e^{i\omega t} d\Theta(z, \omega) dz
\] (25)

where
\[
\tilde{G}^{(p)}(z, \omega, t) = e^{-i\omega t} \frac{\partial^p}{\partial \omega^p} [G(z, \omega, t) e^{i\omega t}] \] (26)
is \( e^{i\omega t} \) multiplied by the \( p \)th time derivative of the fundamental mode response to the excitation \( A(z, \omega, t) e^{i\omega t} \).

The cross-correlation function and the cross-spectrum between the \( p \)th and \( f \)th time derivatives of the generalized displacement, i.e., \( q^{(p)}(t) \) and \( q^{(f)}(t) \), are given as
\[
R_{q^{(p)}q^{(f)}}(t, \tau) = \int_{0}^{H} \int_{0}^{H} \tilde{G}^{(p)}(\omega, t) \tilde{G}^{(f)}(\omega, t) \times (\omega, t + \tau) S_u(z_1, z_2, \omega) e^{i\omega t} dz_1 dz_2 d\omega
\] (27)
\[
S_{q^{(p)}q^{(f)}}(\omega, t) = \int_{0}^{H} \int_{0}^{H} \tilde{G}^{(p)}(\omega, t) \tilde{G}^{(f)}(\omega, t) S_u(z_1, z_2, \omega) dz_1 dz_2
\] (28)

### Applications to Some Simplified Cases

Consider the applications of the aforementioned general framework to some simplified cases. It is assumed that the time varying mean wind speed is characterized by a time-invariant vertical profile \( \tilde{U}_d(z) \) and a time function \( d(t) \) as
\[
\tilde{U}(z, t) = \tilde{U}_d(z)d(t)
\] (29)

The wind fluctuation \( u'(z, t) \) is assumed to be a uniformly modulated evolutionary process with the modulation function identical to the time function for the mean wind speed, i.e., \( g(z, \omega, t) = d(t) \). Subsequently, \( u'(z, t) = u(z, t)d(t) \), where the stationary process \( u(z, t) \) is characterized by its PSD and cross-spectrum as
\[
S_u(z, \omega) = S_{u0}(\omega) I(z)
\] (30)
\[
S_u(z, z, \omega) = \sqrt{S_u(z, z, \omega) \sqrt{S_u(z, z, \omega) \sqrt{S_u(z, z, \omega)}}}
\] (31)
\[
\text{coh}(z, z, \omega) = \exp \left( -\frac{k \omega}{z - z_0} \right)
\] (32)

where \( S_{u0}(\omega) = \) PSD function; \( I(z) = \) function of elevation \( z \), which influences the vertical profile of the turbulence intensity of \( u'(z, t) \) and \( u(z, t) \) as \( \sigma_u(z, \omega) / \tilde{U}_d(z) \) where \( \sigma_u^2 = \int_0^\infty S_{u0}(\omega) d\omega \); \( \text{coh}(z, z, \omega) = \) coherence function; \( k \) = decay factor; and \( \tilde{U}_d = \) reference wind speed, taken as the maximum mean wind speed over the building height.

Following the aforementioned general framework and denoting \( A_p(t) = \tilde{d}^2(t) \), the generalized displacement in terms of \( \tilde{q}(t) \) and the EPSD of \( q'(t) \) are determined as
\[
\tilde{q}(t) = \tilde{q}_0 \tilde{A}_0 \tilde{G}_0(0, t)
\] (33)
\[
S_{q'}(\omega, t) = \left| G_0(\omega, t) \right|^2 I_2(\omega) S_{p0}(\omega)
\] (34)

where
\[
q_R = 0.5pC_p U_R^2 RH
\] (35)

\[
\tilde{A}_0 = \int_{|z|}^{H} \left( \frac{z}{H} \right)^\mu \tilde{U}_d^2(z) d\mu
\] (36)

\[
G_0(\omega, t) = \int_{0}^{H} h(t - \tau) A_P(\tau) e^{i\omega t} d\tau
\] (37)
\[
S_{p0}(\omega) = 4q_R^2 |G_0(\omega)|^2 S_{u0}(\omega) \sigma_u^2
\] (38)
and $I_{B} = \sigma_{w} / U_{R}$=turbulence intensity.

Similarly, the cross-spectrum $S_{q^2(p^2)}(\omega)$ is expressed as

$$S_{q^2(p^2)}(\omega) = G_{0}(\omega,t) G_{0}^{*}(\omega,t) |J_{1}(\omega)|^{2} \tilde{S}_{p}(\omega)$$

where

$$G_{0}^{*}(\omega,t) = e^{-i\omega t} \frac{\partial}{\partial t} [G_{0}(\omega,t) e^{i\omega t}]$$

It is noteworthy that $G_{0}(\omega,t)$ and $G_{0}^{*}(\omega,t)$ can be interpreted as $e^{-i\omega t}$, respectively, multiplied by the fundamental mode response to the excitation $A_{f}(t) e^{i\omega t}$ and its $p$th time derivative.

When the variation rate of $A_{f}(t) = d^{2}(t)$ is very small as compared to the structural modal frequency, and $t$ is sufficiently large so that $e^{-\xi t}$ approaches to zero, $G_{0}(\omega,t)$ and $G_{0}^{*}(\omega,t)$ can be approximated as

$$G_{0}(\omega,t) = A_{f}(t) \int_{0}^{t} h(\tau) e^{-i\omega \tau} d\tau$$

$$= A_{f}(t) \int_{0}^{\infty} h(\tau) e^{-i\omega \tau} d\tau$$

$$= A_{f}(t) H(\omega)$$

$$G_{0}^{*}(\omega,t) = A_{f}(t)(i\omega) H(\omega)$$

$$H(\omega) = 1/[m_{1}(-\omega^{2} + 2i\xi_{0} \omega + \omega_{0}^{2})]^{-1}$$

where $H(\omega)$ is the frequency response function of the fundamental mode. Subsequently

$$\bar{q}(t) = q_{R} \tilde{A}_{0} A_{f}(t)/K_{1}$$

$$S_{q^2(p^2)}(\omega,t) = A_{f}^{2}(t) |H(\omega)|^{2} |J_{1}(\omega)|^{2} \tilde{S}_{p}(\omega)$$

$$S_{q^2(p^2)}(\omega,t) = A_{f}^{2}(t)(-i\omega)(i\omega) H(\omega)^{2} |J_{1}(\omega)|^{2} \tilde{S}_{p}(\omega)$$

that indicate that the effects of transient structural dynamics are negligible. The building response varies with time following the modulation function $A_{f}(t)$, i.e., building responds to the transient wind quasi-statically, thus, referred to as “quasi-static” response. In such a special case, the response analysis at each time instant is the same as that for the traditional stationary winds. It is also noted that when $A_{f}(t) = 1$, these formulations reduce to those for the traditional stationary boundary layer winds with constant mean and stationary random wind fluctuations (Piccardo and Solari 2000; Chen and Kareem 2004).

From Eqs. (33) to (39), it is clear that the influence of wind speed vertical profile on the mean and RMS building response is reflected by the values of the constant $\tilde{A}_{0}$ and the joint acceptance function $|J_{1}(\omega)|^{2}$, respectively. The joint acceptance function also involves the reduction effect due to the lack of spatial correlation in wind loads. The effects of time varying mean and evolutionary characteristics of fluctuating wind speed on building response are reflected by the functions $G_{0}(0,t)$ and $G_{0}(\omega,t)$.

In the case of a power law profile
layer winds with power law exponents $\alpha=0.1,0.2,$ and 0.3. For buildings with a linear mode shape, i.e., $\beta=1$, the corresponding values of $A_0$ in the cases of downburst wind profiles with $H/\delta=0.25,0.5,0.75,$ and 1 are 0.4863, 0.4434, 0.3593, and 0.2768, respectively. On the other hand, in the cases of the power law wind profiles with $\alpha=0.1,0.2,$ and 0.3, they are 0.4545, 0.4167, and 0.3846. Fig. 2 shows $[J_z(\omega)]^2$ as a function of both $H/\delta$ and $k_0\omega H/(2\pi U_R)=k_0 f H/U_R$ with $I(z)=1$. It is noted that, under the condition that the maximum wind speeds over the building height are identical, the downburst profile gives a similar wind load effect for lower buildings with $H/\delta$ less than around 0.5, but a smaller wind load effect for taller buildings with $H/\delta$ larger than around 0.5 as compared to the traditional power law profiles. Similar discussion can also be made with respect to the wind speed on building response, five different modulation functions in term of $A_F(t)=F(t)$ are considered, as shown in Fig. 4. These include two functions given by the three-parameter function

$$A_F(t) = \alpha_0 \beta_0 \gamma e^{-\gamma t} \quad \alpha_0 > 0 \quad \beta_0, \gamma \geq 0$$

with a maximum value of unity at the time instant $t_{\text{max}}=\beta_0/\gamma$ where $\alpha_0=\lambda^{H_0}/\beta_0^{\lambda^{H_0}}$. These two wind events, referred to as “Wind event 1” and “Wind event 2”, correspond to $A_F(t)$ with $\beta_0=2$ and $t_{\text{max}}=60$ and 120 s, respectively. For these three-parameter modulation functions, $G_1(\omega,t)$ can be determined by a closed-form formulation for a nonnegative integer $\beta_0$ [see (Conte and Peng 1997) and the Appendix]. Another modulation function, corresponding to “Wind event 3,” is numerically simulated based on the empirical model suggested by Holmes and Oliver (2000) as applied to Andrews A.F.B. downburst. This model describes the horizontal wind speed in a traveling downburst generated by the vector summation of the translation speed and the radial wind generated by an impinging jet. A modification of this model has also been proposed by Chay et al. (2006). The remaining two modulation functions, corresponding to “Wind event 4” and “Wind event 5,” are directly obtained from the full-scale data of the outflow of a rear-flank downburst (RFD) recorded on June 4, 2002, and a convective windstorm Derecho recorded on June 15, 2002, both at the Reese Technology Center, Texas Tech University (Gast and Schroeder 2003).

To study the influence of wind speed profile on building response, the power law profile with an exponent $\alpha=0.25$ and the downburst wind profiles as suggested by Wood et al. (2001) with $H/\delta=0.5$ and 1 are considered in the analysis. For each case, it is assumed that $I(z)=1$, i.e., $S_0(z,\omega) S_0(\omega)$, and $S_0(f) = 4\pi S_0(\omega)$ (where $\omega=2\pi f$) is given by von Karman spectrum

$$fS_0(f) = \frac{4/L_w^2 U_R}{\left[1+70.78(U_R/L_w)^{2}\right]^{5/6}}$$

where $L_w$ is the integral length scale taken as 80 m, and the turbulence intensity $I_w=\sigma_0^2/U_R$ is 15%. For the purpose of comparison, both partially and fully correlated wind loads over the building height are considered, which correspond to the decay factor $k_s$ as 8 and 0, respectively, in the coherence function of wind fluctuations, i.e., $\text{coh}(z_1, z_2, \omega)$. As will be illustrated, because the building response is a narrow-band process, the selection of wind speed and wind load characteristics does not influence the general conclusion concerning the fundamental feature of building response to transient winds.
Fig. 4. Time modulation functions of the mean wind speeds (a) Wind events 1 and 2; (b) Wind event 3; (c) Wind event 4; and (d) Wind event 5.

Fig. 5. Mean and RMS values of building tip displacement ($H=200$ m, $\xi_1=1\%$, $\alpha=0.25$, $U_H=40$ m/s) (a) Wind event 1; (b) Wind event 3; (c) Wind event 4; and (d) Wind event 5.
Fig. 5 shows the time varying mean and RMS values of the tip displacement of the building with \( H = 200 \) m under different wind events of consideration. All these wind events are assumed to have an identical power law mean wind speed profile with the exponent \( \alpha = 0.25 \) and the maximum mean wind speed at the building height \( U_H = 40 \) m/s. The displacement is normalized by a factor of \( q_R/K_1 \), i.e., static building displacement under load \( q_R \). As only the fundamental mode contribution is involved in the analysis, all building response components, e.g., base bending moment and base shear, have the same characteristics as the building tip displacement. The corresponding quasi-static responses in which the building is assumed to respond to transient winds quasi-statically are also shown in Fig. 5. The closed-form formulations were used for Wind events 1 and 2, while the linear acceleration step-by-step integration method was utilized for Wind events 3, 4, and 5. It is noted that the maximum value of the RMS response in all cases is slightly delayed with respect to the time at which the mean wind speed reaches its maximum. This time lag phenomenon has also been reported by Solomos and Spanos (1984), Igusa (1989a,b), and Conte and Peng (1997) in dealing with structural response to nonstationary white noise excitation.

It is also emphasized that the maximum value of RMS response is smaller than that of the quasi-static counterpart. It is due to the lack of sufficient “build-up” time to reach the steady-state response as predicted by the quasi-static analysis. The “build-up” time is the time required to make \( e^{-2\xi_1\omega_1 t} \) close to zero. Clearly, a large value of \( \xi_1\omega_1 \), a shorter “build-up” time needed, and the RMS response becomes closer to the quasi-static value. This feature is clearly demonstrated in Fig. 6, which compares the RMS responses for different buildings with different frequencies and different levels of modal damping ratios under Wind events 1 and 2 with a power law mean wind speed profile. For example, of the tall building of 200 m high with a modal frequency of 0.23 Hz and a damping ratio of 1%, the maximum RMS response reaches only 85% of the maximum quasi-static response in the case of “Wind event 1,” 94% in the case of “Wind event 2.” When the damping ratio increases to 2%, these become 94 and 98%, respectively. The difference between the predicted RMS response and quasi-static value increases with decreasing natural frequency. The spatial correlation of wind loads does not influence the response “build-up,” while it significantly affects the value of building response. Fig. 7 shows the tip displacement and acceleration of the building of 200 m high with a damping ratio of 1% under Wind event 1 with increasing wind speed. The acceleration is normalized by a factor of \( q_R/m_1 \), i.e., pseudoacceleration of the building under load \( q_R \). The difference between the predicted RMS response and quasi-static value increases with increasing wind speed.

Tables 1 and 2 summarize the building tip displacement and acceleration of the building of 200 m high with a damping ratio of 1% when subjected to different transient winds. Among these five wind events, Wind events 2 and 5 lead to the largest and smallest response, respectively. As expected, the building response is strongly influenced by the spatial correlation of wind loads, i.e., a stronger correlation of wind loads results in a larger building response.

Tables 3 and 4 summarize the building response to transient winds with downburst wind profiles having \( H/\delta = 0.5 \) and 1.0, respectively. The building modal damping was 1%. The comparison of these results with those shown in Table 1 provides information concerning the influence of wind speed profile on building response. These results are consistent with the values of \( \bar{A}_0 \) and \( |J_a(\omega)|^2 \) as shown in Fig. 2, which reflect the effect of wind speed profile on building response. For example, the profile with \( H/\delta = 1 \) results in a smaller building response as compared to the profile with \( H/\delta = 0.5 \).

**Concluding Remarks**

This study presented a frequency domain analytical approach for quantifying alongwind tall building response when subjected to nonstationary winds. The transient wind fluctuations and associated wind loads were modeled as the sum of deterministic time varying mean and evolutionary random fluctuation components. This approach provided estimations of the time varying mean, evolutionary spectrum, and time varying root-mean-square value of building response for a given nonstationary wind characterized by its vertical profile, modulation function, and turbulence characteristics. Based on this approach, the influence of time varying mean wind speed, mean wind speed vertical profile, and spatial correlation of wind loads on building response was addressed using a number of examples that include thunderstorm downburst
winds recorded from full-scale measurements. The transient nature of wind speed generally leads to lower response as the result of the lack of sufficient “build-up” time to reach its steady-state value. This “building-up” effect depends on the damping ratio, modal frequency, and the variation rate of the mean wind speed. The downburst wind profile resulted in a similar load effect on lower buildings but a smaller effect on taller buildings as compared to the conventional power law profiles. As expected, a stronger spatial correlation of wind loads led to a higher building response.

It is worth mentioning that a complete modeling and quantification of building response to transient wind loads should also consider the effects of the transient aerodynamics, i.e., the potential changes in the wind load characteristics, e.g., admittance and coherence functions, associated with transient winds. The effects of transient aerodynamics may be particularly significant when the variation rate of wind speed is large as compared to the time scale of the generation of wind loads. In this regard, a comprehensive wind tunnel study under adequately simulated transient winds will be required to better characterize and model wind loads on buildings under transient winds. This updated loading information can be readily synthesized with the proposed approach of this study, which leads to better prediction of building response when subjected to transient nonstationary winds.

**Acknowledgments**

The support of the new faculty startup funds provided by Texas Tech University is gratefully acknowledged.

**Appendix. Closed-Form Formulation for \( G_0(f, t) \)**

When \( A_F(t)=d^2(t) \) is described by the three-parameter time modulation function

\[
A_F(t) = \alpha e^{\beta t} e^{-\lambda t}, \quad \alpha > 0, \quad \beta, \lambda \geq 0
\]

the integration

\[
G_0(\omega, t) = \int_0^t h(t-\tau) d^2(\tau) e^{-i\omega(t-\tau)} d\tau
\]

can be determined by the following closed-form formula:

\[
G_0(\omega, t) = \frac{j\tilde{x}_0}{2m\omega_D} \left[ e^{-\lambda t} \sum_{n=0}^{\infty} E_n(-E_{e1} + E_{e2}) + E_n(E_{eb1} - E_{eb2}) \right]
\]

where

\[
E_n(n, \beta, \theta) = (-1)^n \beta (\theta - n)!
\]

\[
E_{e1}(n, \theta, \rho) = \exp[-i(n+1)(\theta + \rho)]
\]
Table 3. Influence of Wind Speed Profiles on Building Tip Displacement ($H = 200 \text{ m}, U_{\text{max}} = 40 \text{ m/s}$)

<table>
<thead>
<tr>
<th></th>
<th>$H/\delta = 0.5$</th>
<th>$H/\delta = 1$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Wind event 1</td>
<td>Wind event 2</td>
</tr>
<tr>
<td>Mean RMS</td>
<td>0.4438</td>
<td>0.4434</td>
</tr>
<tr>
<td>RMS (quasi-static)</td>
<td>0.1694</td>
<td>0.1911</td>
</tr>
</tbody>
</table>

Table 4. Influence of Wind Speed Profiles on Building Tip Acceleration ($H = 200 \text{ m}, U_{\text{max}} = 40 \text{ m/s}$)

<table>
<thead>
<tr>
<th></th>
<th>$H/\delta = 0.5$</th>
<th>$H/\delta = 1$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Wind event 1</td>
<td>Wind event 2</td>
</tr>
<tr>
<td>RMS RMS</td>
<td>0.2010</td>
<td>0.2010</td>
</tr>
</tbody>
</table>

References


